HYDROLOGIC EFFECTS OF HYPOTHETICAL EARTHQUAKE-CAUSED FLOODS BELOW JACKSON LAKE, NORTHWESTERN WYOMING

By William R. Glass, Thomas N. Keefer, and James G. Rankl

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CONTENTS

ILLUSTRATIONS

Page

TABLE

Page

CONVERSION FACTORS

Factors for converting English units to metric units are shown to four significant figures. However, in the text the metric equivalents are shown only to the number of significant figures consistent with the values for the English units.

English	Multiply by	Metric
acre-ft (acre-feet)	1.233×10^{-3}	hm^3 (cubic hectometres)
ft^3/s (cubic feet per second)	2.832×10^{-2}	m^3/s (cubic feet per second)
ft/s (feet per second)	3.048×10^{-1}	m/s (metres per second)
ft (feet)	3.048×10^{-1}	m (metres)
mi (miles)	1.609	km (kilometres)

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ABSTRACT

Jackson Lake, located in Grand Teton National Park, Wyoming, is in an area of seismic instability. There is a possibility of flooding in the Snake River downstream from Jackson Lake Dam in the event of a severe earthquake. It is not possible to determine the probability of such an event. Hypothetical floods were routed 38 miles (61 kilometres) downstream from the dam for three cases: 1) Instantaneous destruction of the dam outlet structure, 2) instantaneous destruction of the entire dam, and 3) for waves overtopping the dam without failure of the dam. In each case, a full reservoir was assumed.

Hydrographs for outflow from the reservoir for the two cases of dam failure were developed utilizing an accelerated discharge due to the travel of a negative wave through the reservoir, and Muskingum storage routing. For the case of waves overtopping the dam, a 10-foot (3-metre) wave was assumed to be propagated from the upstream end of the reservoir. A multiple-linearization technique was used to route the flow through the reach. The model was calibrated from U.S. Geological Survey streamflow records of inflow and outflow in the study reach. Attenuation and damping of the outflow hydrograph from the lake required development of width-discharge and celerity-discharge relations for cross sections located in the study reach. The peak discharges calculated as outflow from Jackson Lake were 192,600; 532,500; and 350,000 ft^3/s (cubic feet per second) or 5,450; 15,070; and 9,910 m^3/s (cubic metres per second), respectively, for the three cases. These discharges, according to the modeled results, attenuated 30 miles (48 kilometres) downstream to 184,400; 448,900; and 55,100 ft^3/s (5,220; 12,710; and 1,560 m^3/s). Tributary inflow used for this reach was 11,500 ft^3/s (330 m^3/s).

An inundation map was prepared from channel conveyance curves and profiles of the water surface.

Flooding would be relatively confined upstream from the settlement at Moose, but would become more widespread downstream where the valley becomes wider. Most extensive flooding and greatest water velocities would occur if the entire dam were destroyed; floods for the other two cases were smaller.

Effects of these flows would include extensive scour and fill, changes in position of the channel, destruction of buildings and flood-plain vegetation, and possible contamination of wells by silt and bacteria.

INTRODUCTION

This report deals with the hypothetical sudden catastrophic release of water from Jackson Lake that might occur as the result of an earthquake. The study reach of the Snake River covers approximately 38 mi (61 km), from Jackson Lake Dam, in Grand Teton National Park, to latitude 43°26'30", a point about 3.5 mi (5.6 km) south of Wilson, Wyoming (fig. 1).

The study was made in cooperation with the National Park Service to aid in location of future park facilities. The purposes of the study were to prepare a flood-inundation map and to evaluate consequences and characteristics of possible floodflows.

Jackson Lake is situated in an area of seismic instability, with the Teton Fault zone running through the western part of the lake. Two cases of geologic instability that had associated historical hydrologic events in the area were: 1) The Lower Gros Ventre Slide across the Gros Ventre River, which was washed out on June 23, 1925, releasing about 60,000 acre-ft (74 hm^3), of impounded water; and 2) the earthquake in Yellowstone National Park, August 17, 1959, which caused waves that overtopped the Hebgen Lake Dam (U.S. Geological Survey, 1964).

In this study, floods were routed for two cases of failure of Jackson Lake Dam and for the case in which waves overtop the dam without dam failure. The reservoir was assumed full for all cases. The two cases of dam failure used were: 1) Instantaneous destruction of the concrete outlet portion of the dam, and 2) instantaneous destruction of the entire dam. The two cases of dam failure were selected not on a basis of most likely occurrence but rather to give a range of discharge up to the worst conceivable disaster, that is, the entire dam destroyed (case 2). Balloffet and others (1974) assumed a rectangular gap dam breach, whose width increased uniformly with time, in their routing of a dam-collapse flood wave in Morocco. It was felt little would be gained in this study by a more elaborate dam-collapse mechanism, because peak flows are of greatest interest and would be sustained by the large capacity of Jackson Lake, 847,000 acre-ft (1,044 hm^3).

Floodflow also could originate from Bearpaw Bay of Jackson Lake and flow toward Moose if significant slippage in the Teton Fault zone occurred. This condition was not considered in the analysis.

The flood inundation map for the three hypothetical floodflows was defined by the following steps: 1) Hydrographs of outflow from the reservoir for the two cases of dam failure were developed utilizing an accelerated discharge due to the travel of a negative wave through the reservoir, and Muskingham storage routing. The hydrograph for the case of waves overtopping the dam was arrived at similarly except a 10-ft (3-m) wave was assumed to be propagated from the upstream end of the reservoir; 2) a multiple-linearization technique was used to route the flow through the reach. The model, calibrated by records of streamflow

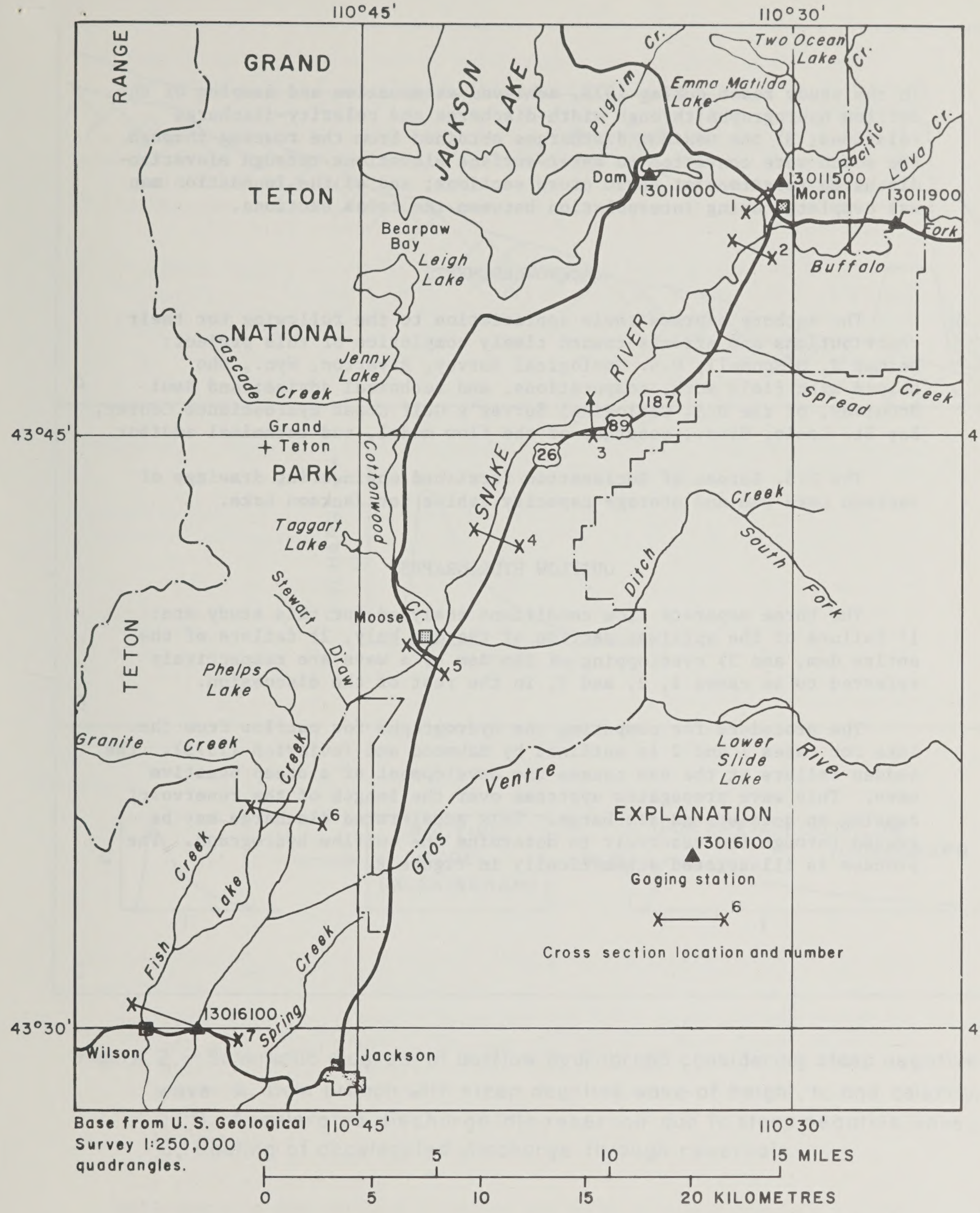


Figure 1.— Jackson Lake-Snake River study area.

in the study reach during 1973, achieved attenuation and damping of the outflow hydrographs through width-discharge and celerity-discharge relations; 3) the maximum discharges obtained from the routing through the reach were converted to water-surface elevations through elevation-discharge relations at seven cross sections; and 4) the inundation map was completed using interpolation between the cross sections.

ACKNOWLEDGMENTS

The authors express their appreciation to the following for their contributions and efforts toward timely completion of this project: Delmer J. O'Connell, U.S. Geological Survey, Riverton, Wyo., who helped with field work, computations, and technical advice; and Raul McQuivey, of the U.S. Geological Survey's Gulf Coast Hydroscience Center, Bay St. Louis, Miss., coauthor of the flow model, and technical advisor.

The U.S. Bureau of Reclamation furnished engineering drawings of Jackson Lake Dam and storage capacity tables for Jackson Lake.

OUTFLOW HYDROGRAPHS

The three separate flow conditions analyzed for this study are: 1) Failure of the spillway section of the dam only, 2) failure of the entire dam, and 3) overtopping of the dam by a wave are respectively referred to as cases 1, 2, and 3, in the rest of the discussion.

The procedure for computing the hydrographs for outflow from the lake for cases 1 and 2 is outlined by Mahmood and Yevjevich (1975). The sudden failure at the dam causes the development of a steep negative wave. This wave propagates upstream over the length of the reservoir, causing an accelerated discharge. This accelerated discharge may be routed through the reservoir to determine the outflow hydrograph. The process is illustrated schematically in figure 2.

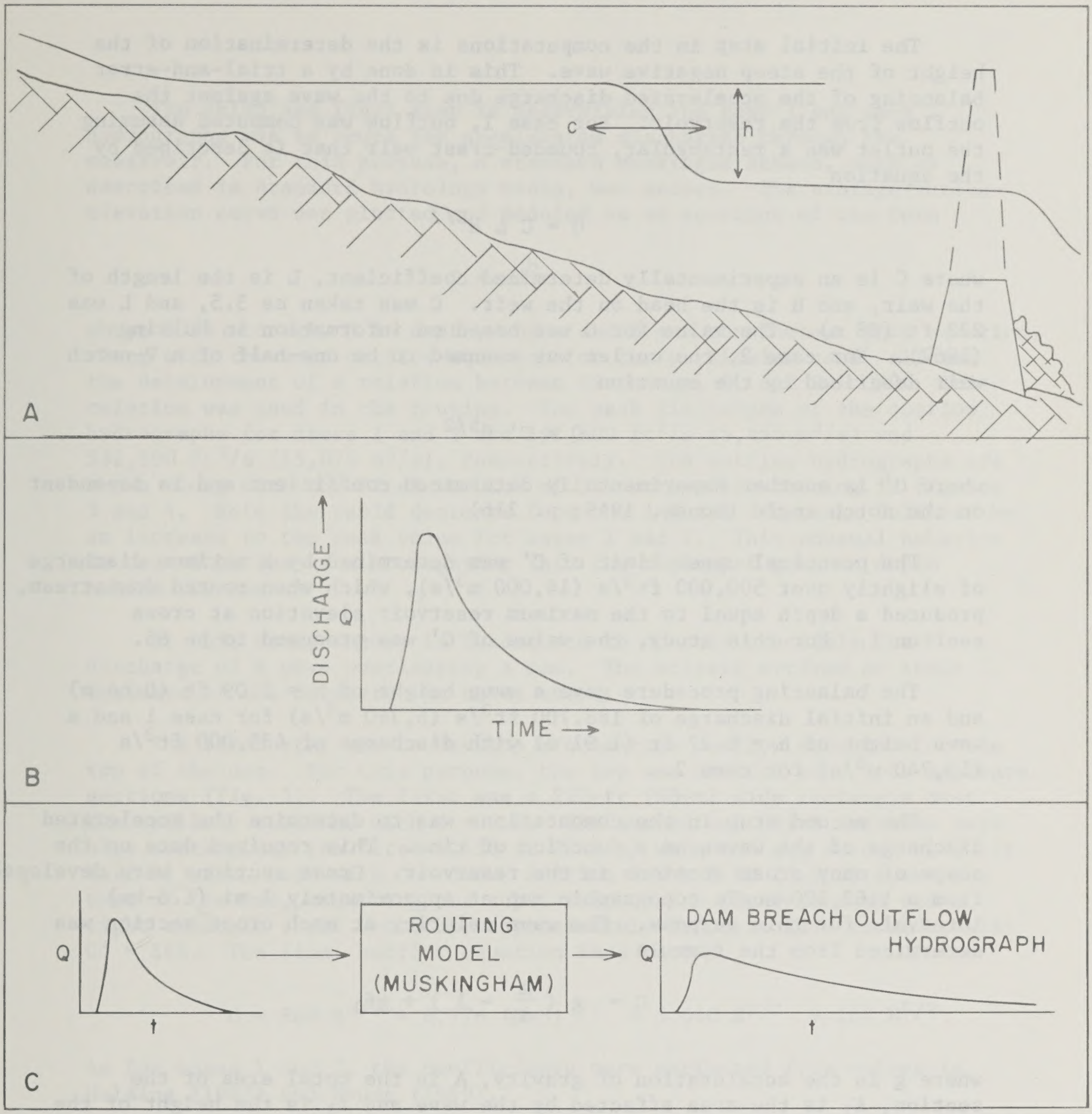


Figure 2.—Schematic diagram of outflow hydrograph considering steep negative wave: A, Dam breach with steep negative wave of height, h, and celerity, c; B, Accelerated discharge into reservoir due to steep negative wave; C, Routing of accelerated discharge through reservoir.

The initial step in the computations is the determination of the height of the steep negative wave. This is done by a trial-and-error balancing of the accelerated discharge due to the wave against the outflow from the reservoir. For case 1, outflow was computed assuming the outlet was a rectangular, rounded-crest weir that is described by the equation

$$Q = C\ L\ H^{3/2}$$

where C is an experimentally determined coefficient, L is the length of the weir, and H is the head on the weir. C was taken as 3.5, and L was 222 ft (68 m). The value for C was based on information in Hulsing (1967). For case 2, the outlet was assumed to be one-half of a V-notch weir described by the equation

$$Q = C'\ H^{5/2}$$

where C' is another experimentally determined coefficient and is dependent on the notch angle (Rouse, 1949; p. 216).

The practical upper limit of C' was determined by a maximum discharge of slightly over 500,000 ft^3/s (14,000 m^3/s), which when routed downstream, produced a depth equal to the maximum reservoir elevation at cross section 1. For this study, the value of C' was presumed to be 65.

The balancing procedure gave a wave height of h = 2.09 ft (0.64 m) and an initial discharge of 188,700 ft^3/s (5,340 m^3/s) for case 1 and a wave height of h = 6.27 ft (1.91 m) with discharge of 485,000 ft^3/s (13,740 m^3/s) for case 2.

The second step in the computations was to determine the accelerated discharge of the waves as a function of time. This required data on the shape of many cross sections in the reservoir. Cross sections were developed from a 1:62,500-scale topographic map at approximately 1-mi (1.6-km) intervals for this purpose. The wave celerity at each cross section was determined from the formula

$$C = g\left(\frac{A}{A_1} - 1\right) + gf_1$$

where g is the acceleration of gravity, A is the total area of the section, A_1 is the area affected by the wave and f_1 is the height of the center of gravity of A_1 (Mahmood and Yevjevich, 1975). Accelerated discharge values were computed as CA_1. The time required for the wave to reach each cross section was computed by the relation

$$t = L \int_x^1 \frac{dx}{C}, \quad \text{where } x = 1 - \frac{X}{L},$$

and X = distance upstream from the outlet, t = time, and L is the total length of the reservoir. The initial velocity in the reservoir was assumed small compared to C.

The third and final step in determining the case 1 and case 2 hydrographs was to route the accelerated discharges through the reservoir. For this purpose, a standard Muskingum scheme, such as described in standard hydrology books, was needed. The storage-versus-elevation curve was plotted and modeled as an equation of the form

$$S = aH^{b}$$

where S is storage, H is depth, and a and b are constants. This equation, along with the weir outflow equation for the particular case, allowed the development of a relation between discharge, Q, and S + Q/2. This relation was used in the routing. The peak discharges of the outflow hydrographs for cases 1 and 2 are 192,600 ft^3/s (5,450 m^3/s) and 532,500 ft^3/s (15,070 m^3/s), respectively. The outflow hydrographs are shown along with the final results of the streamflow routing in figures 3 and 4. Note the rapid decrease from the initial discharge followed by an increase to the peak value for cases 1 and 2. This unusual behavior is caused by the wave propagating into the region of greater depth behind the dam.

At this time, theory is inadequate to compute precisely the discharge of a wave overlapping a dam. The writers arrived at their estimate of the discharge in three steps.

The first step was to develop a weir-type outflow equation for the top of the dam. For this purpose, the top was divided into four separate sections (fig. 5). The first was a 222-ft (68-m) wide rectangle over the spillway section, with C = 4.0. The second and third sections were also rectangles, the first 2,438 ft (743 m) wide and the second 1,400 ft (428 m) wide, both with C = 3.6. Section 2 was 3 ft (0.9 m) above the rest of the dam, requiring the subdivision. The final section was a right triangle (half V-notch), approximately 1,300 ft (396 m) wide, with C' = 168. The final outflow equation thus became

$$Q = 888\ H^{3/2} + 8{,}776\ (H-3)^{3/2} + 5{,}040\ H^{3/2} + 168\ H^{5/2}.$$

As for cases 1 and 2, the coefficients were estimated from values in Hulsing (1967) and Rouse (1949).

The second step was to assume a height of wave and compute the maximum discharge. The wave height was arbitrarily taken as 10 ft (3.0 m) and Q was determined to be 350,000 ft^3/s (9,910 m^3/s).

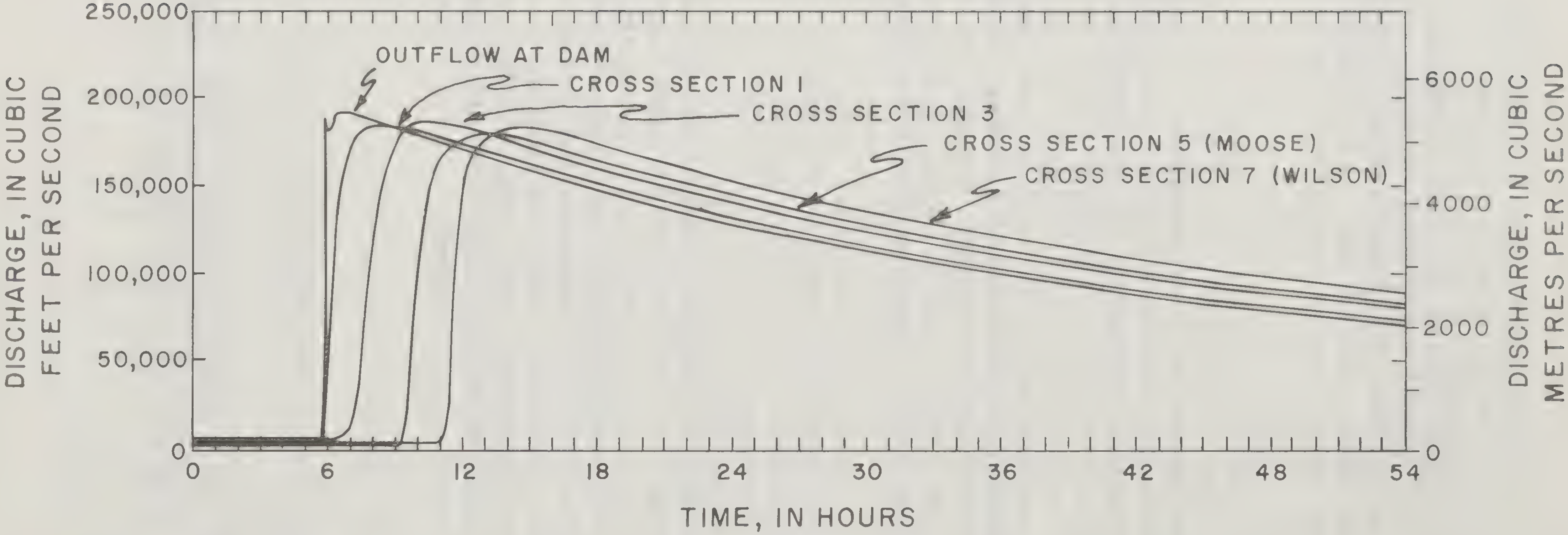


Figure 3.—Outflow hydrograph and the model-routed hydrographs for case I, flood resulting from instantaneous destruction of the outlet portion of Jackson Lake Dam.

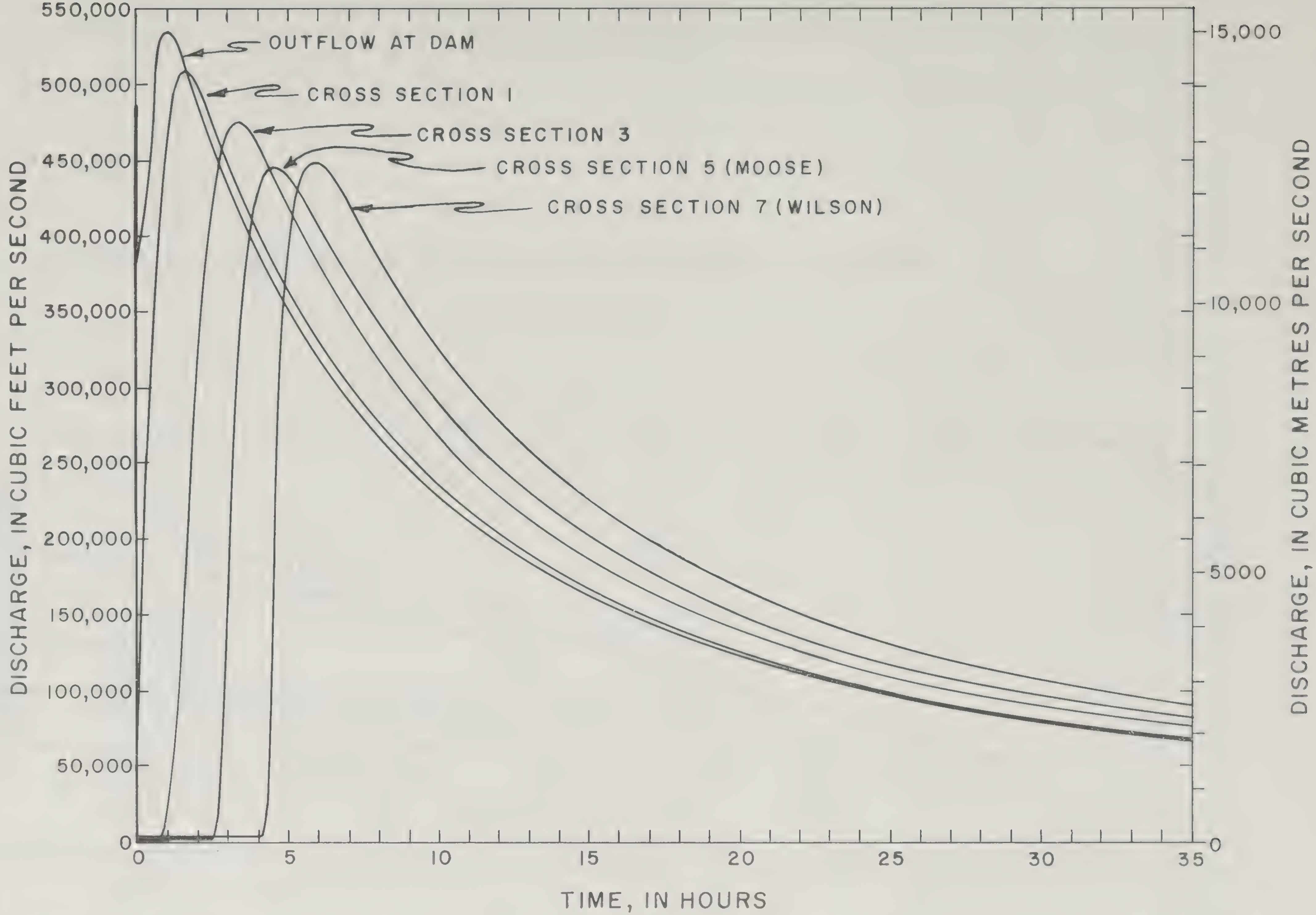


Figure 4.—Outflow hydrograph and the model-routed hydrographs for case 2, flood resulting from instantaneous destruction of all of Jackson Lake Dam.

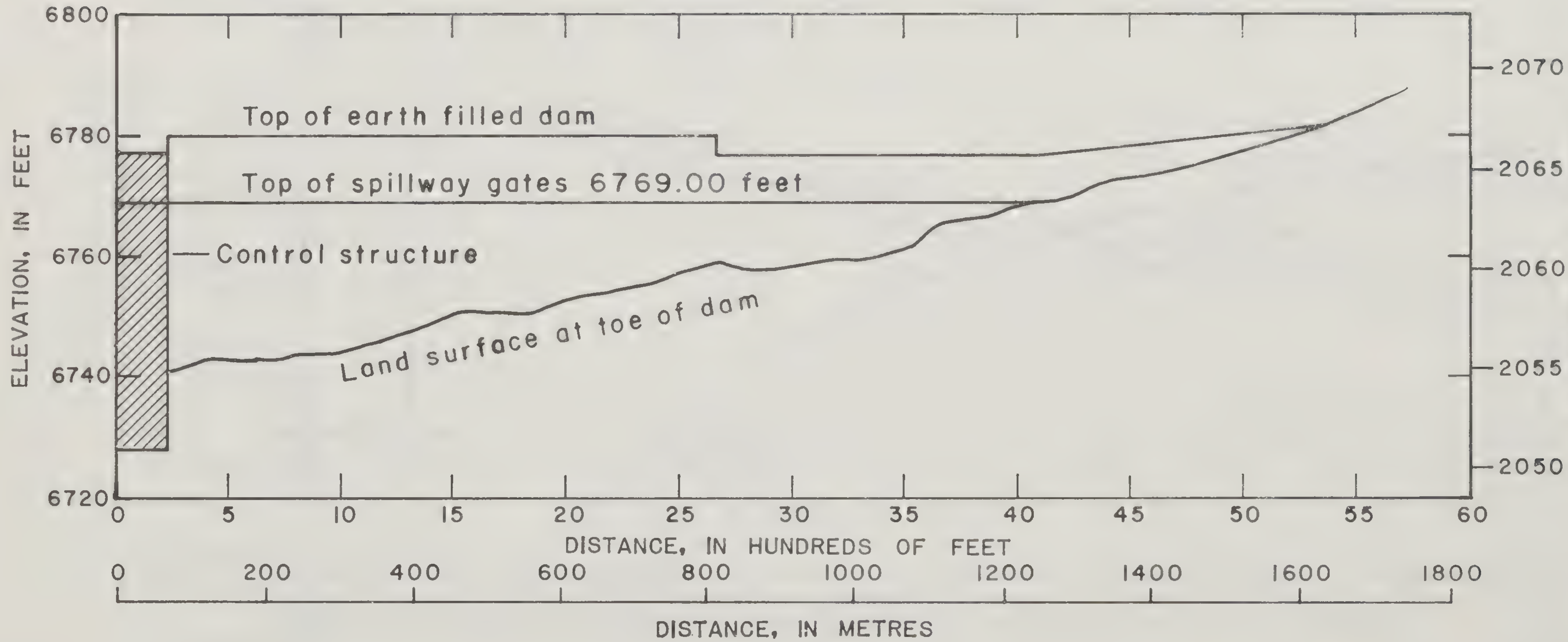


Figure 5.—Cross section of Jackson Lake Dam. Datum is mean sea level.

In the final step it was assumed that the accelerated-discharge hydrographs of cases 1 and 2 would be representative of the shape of a wave-overtopping hydrograph. This should have some merit, in that the shape is highly dependent on the reservoir cross sections, which are reflected in cases 1 and 2. The time frame was assumed to be represented by case 2, except the steep negative wave was assumed to originate at the head of the reservoir, rather than at the dam. Thus, the outflow versus time was determined by: 1) Taking ordinates from the case 2 artificial inflow hydrographs developed from the steep negative wave computations, 2) multiplying them by the ratio of 350,000 divided by the peak value from case 2, and 3) plotting the artificial hydrographs in reverse time order. The result of this procedure is illustrated in figure 6, along with the streamflow routing results.

STREAMFLOW ROUTING MODEL

Description of Model

The model chosen is a multiple linearization technique developed by Keefer and McQuivey (1974). Numerous routing schemes are available, varying in complexity from simple Muskingham storage routing (Linsley and others, 1958), such as used to develop the outflow hydrographs, to complex finite-difference solutions of the complete one-dimensional flow equations as proposed by Amein and Fang (1970). In general, the more complex the model the higher the cost in terms of data and computer time. This is particularly true of accurate finite-difference models, which may require closely spaced frequent cross sections and roughness information for each section along a stream reach. Thus, for many problems, a convolution model that treats the stream reach, however long, as a one-dimensional system, may be of some advantage. These models generate a system response function that is convoluted with the system input to predict the output at a desired point. Multiple linearization is a variation of such models that treat a stream reach as a parallel group of linear systems, each with its own response function. This allows accurate simulation of flood-wave behavior. The concept is illustrated in figure 7.

The data requirements of the model are minimal. In addition to the routing distance and the average slope of the routing reach, all that is required are estimates of the relation between flood-wave celerity and discharge, and the relation between average channel width and discharge.

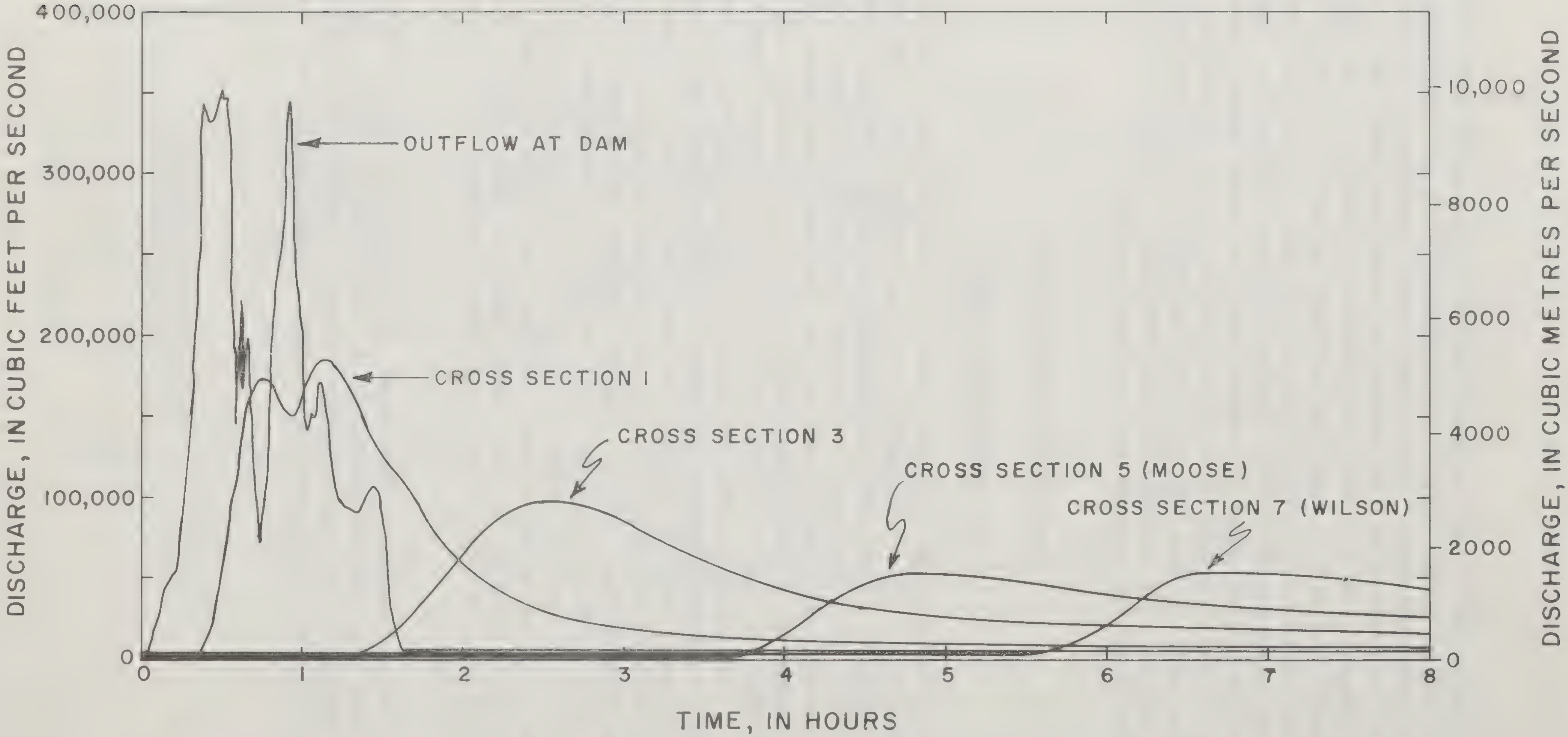


Figure 6.—Outflow hydrograph and the model-routed hydrographs for case 3, flood resulting from waves overtopping Jackson Lake Dam.

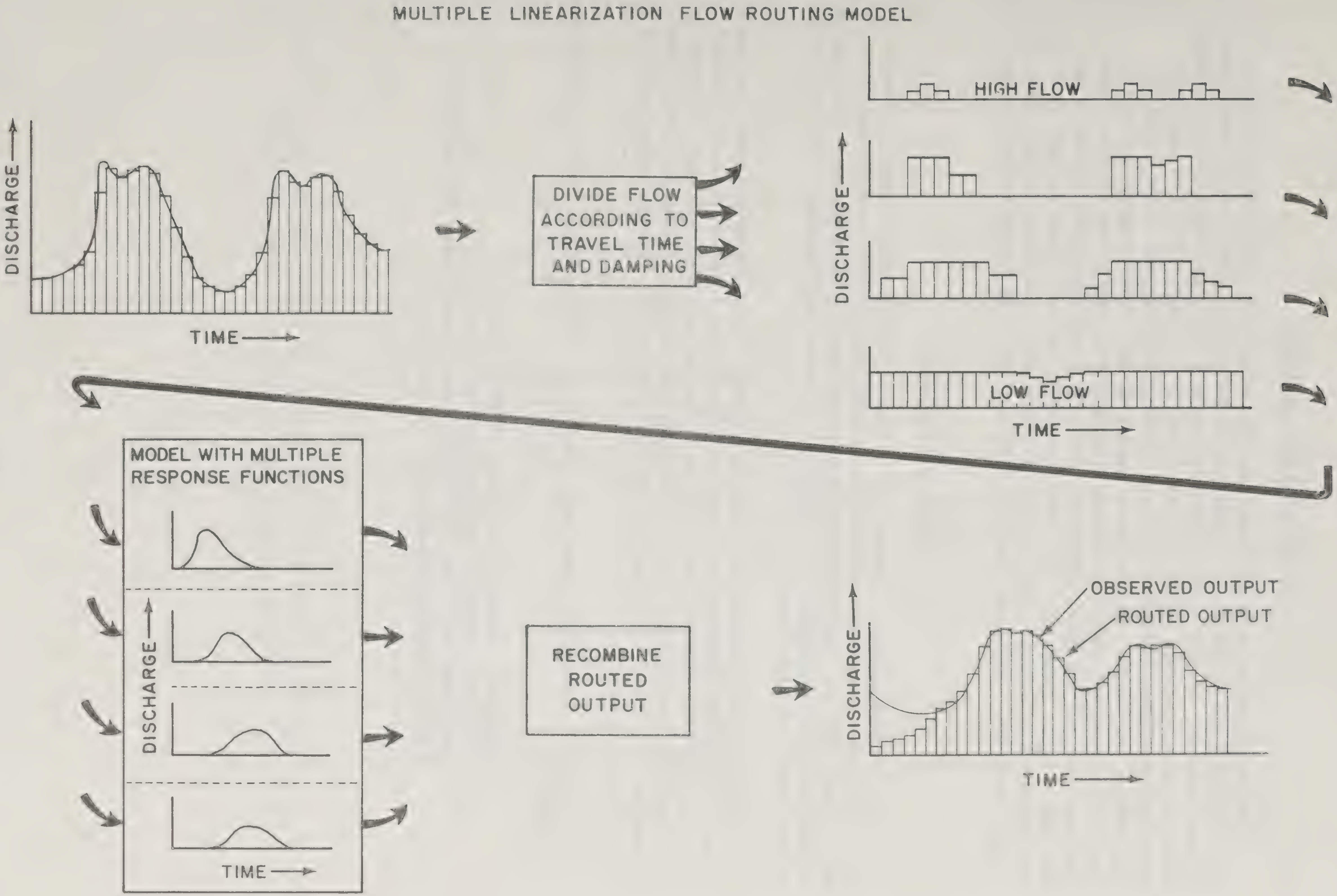


Figure 7.—Diagram of the multiple linearization flow routing model.

Modifications to Model

The original multiple linearization model, as described by Keefer and McQuivey (1974), was not completely suited to flow conditions found in dam breaks. If the travel time through a routing reach was the same order of magnitude as the input data interval, the diffusion analogy overdispersed the predicted output. That is, the peak flow would be suppressed too much. Checks were inserted into the program to insure that the response functions became exponential functions as the travel time approached the data interval. In the limit when the travel time was equal to or less than the data interval, a unit impulse was used as the response function. This is consistent with the behavior of complete linear response functions (Harley, 1967). This is illustrated in figure 8.

Model Calibration

Calibration is the process of adjusting parameters until the model will reproduce historical output with some specified accuracy using historical input. The calibration process serves as a check on the ability of the model to represent the physical process and insures that the parameters used are the "best".

For the study described in this report, only the model used to route flows from below Jackson Lake Dam to Wilson could be calibrated. Only 1973 and 1974 streamflow data for both inflow and outflow were available. The month of June 1973, which contained the highest flows for 1973, was chosen as the first calibration period. Hourly discharges were used as input to the model.

Streamflow records from four gaging stations were used for calibrating the model:

13011000	Snake River near Moran, Wyo.
13011500	Pacific Creek at Moran, Wyo.
13011900	Buffalo Fork above Lava Creek, near Moran, Wyo.
13016100	Snake River near Wilson. Wyo.

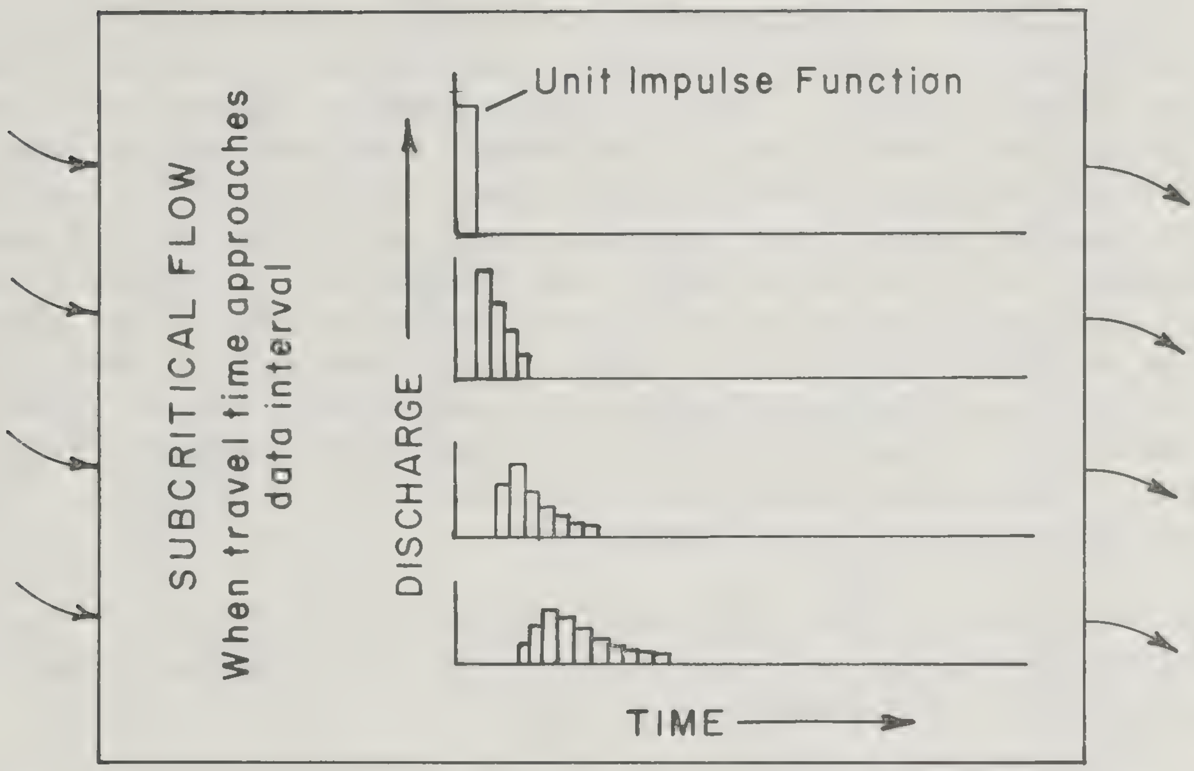


Figure 8.—Diagram of modifications to routing model to handle cases where travel time was less than or equal to the data interval.

The parameters to be determined in the calibration were the wave celerity as a function of discharge, the channel width as a function of discharge, the channel slope, and the routing distance. The routing distance and slope were determined from U.S. Geological Survey 1:24,000-scale topographic maps. No adjustment to these was possible. The width-versus-discharge relation was obtained from the discharge rating tables for cross sections judged to be representative of the routing reaches. The width-versus-discharge relation primarily affects the amount of attenuation, or damping, a wave experiences when traversing a reach. The values used were adjusted by a factor of 2 times the computed value and were found to have little effect on the output. The celerity-versus-discharge relation is more difficult to derive from basic parameters and has a great deal more effect on the model output. For the calibration period, this relation was derived by isolating small flood peaks and determining how long they had taken to traverse the reach. By selecting peaks of various sizes, it was possible to determine accurately a relation for celerity as a function of discharge up to approximately 10,000 ft^3/s (283 m^3/s). Extension of this relation to higher discharges will be discussed later.

For purposes of calibration, the model was run in three steps. The first step was to route the outflow from Jackson Lake to a point 3.8 mi (6.1 km) below the dam. This is approximately the point at which Pacific Creek joins the Snake River. The second step was to add together the routed flow, the flow from Pacific Creek, and the flow from Buffalo Fork, which joins the Snake River approximately half a mile (0.8 km) further downstream. A 2-hour time delay was imposed on the Buffalo Fork data because the gaging station is located approximately 5 mi (8 km) from the junction with the Snake River. The third step was to route the summed flows from the mouth of Buffalo Fork to the gaging station at Wilson (30.14 mi) (48.5 km). To account for the flow from the Gros Ventre River, which is ungaged, 1,300 ft^3/s (36.8 m^3/s) was added to the results. This value was determined by trial and error.

The results of the first calibration are illustrated in figure 9. The model reproduced the historic outflow at Wilson with an error that generally does not exceed 10 percent.

Results of the second, or check, calibration for the period June 1 to June 25, 1974, are illustrated in figure 10. The observed discharge is a preliminary computation and may be revised before publication of the 1974 streamflow records. The river bed scoured more than 1 ft (0.3 m) during the period June 1-25, making determination of discharge difficult. Because the streambed was more stable during June 18-25, observed discharge for that period is probably more accurate than for the period prior to June 18.

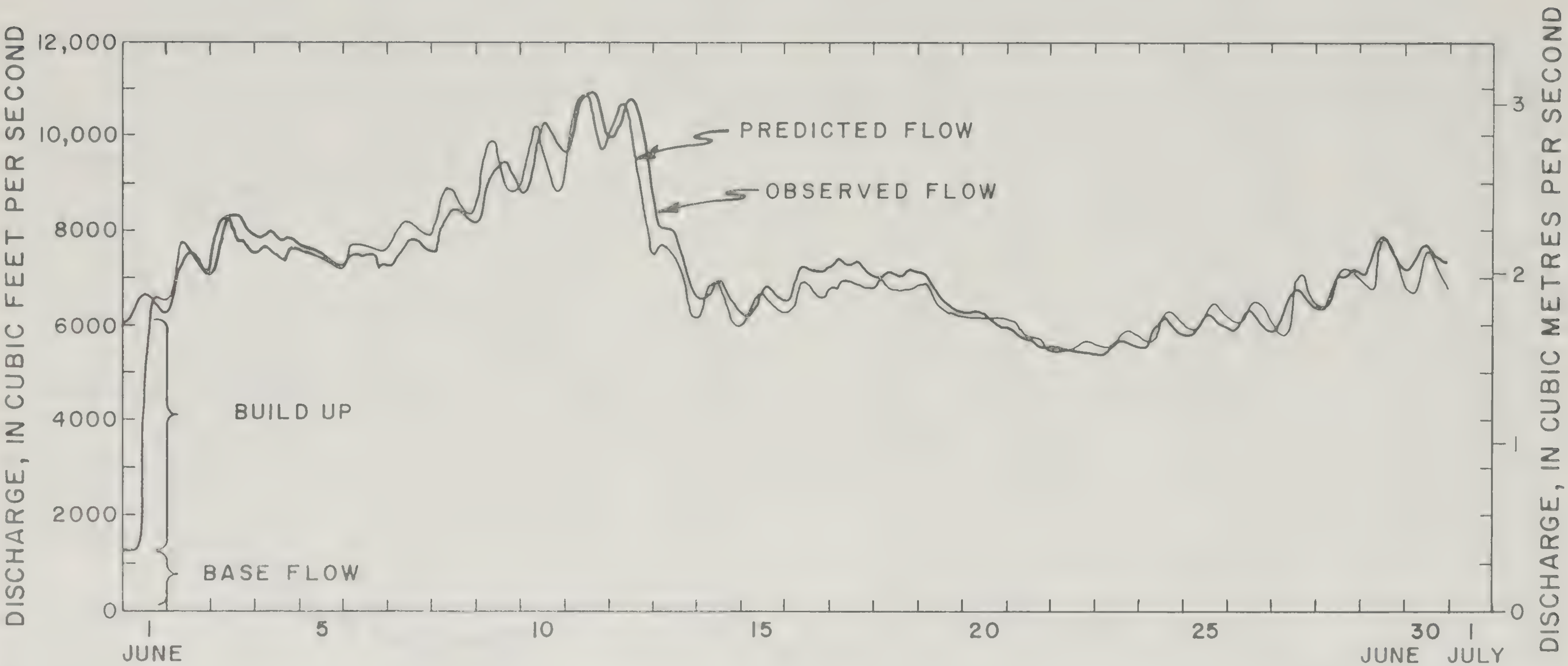


Figure 9.—Observed and modeled discharge hydrographs for the gaging station, Snake River at Wilson, Wyoming, June 1-July 1, 1973.

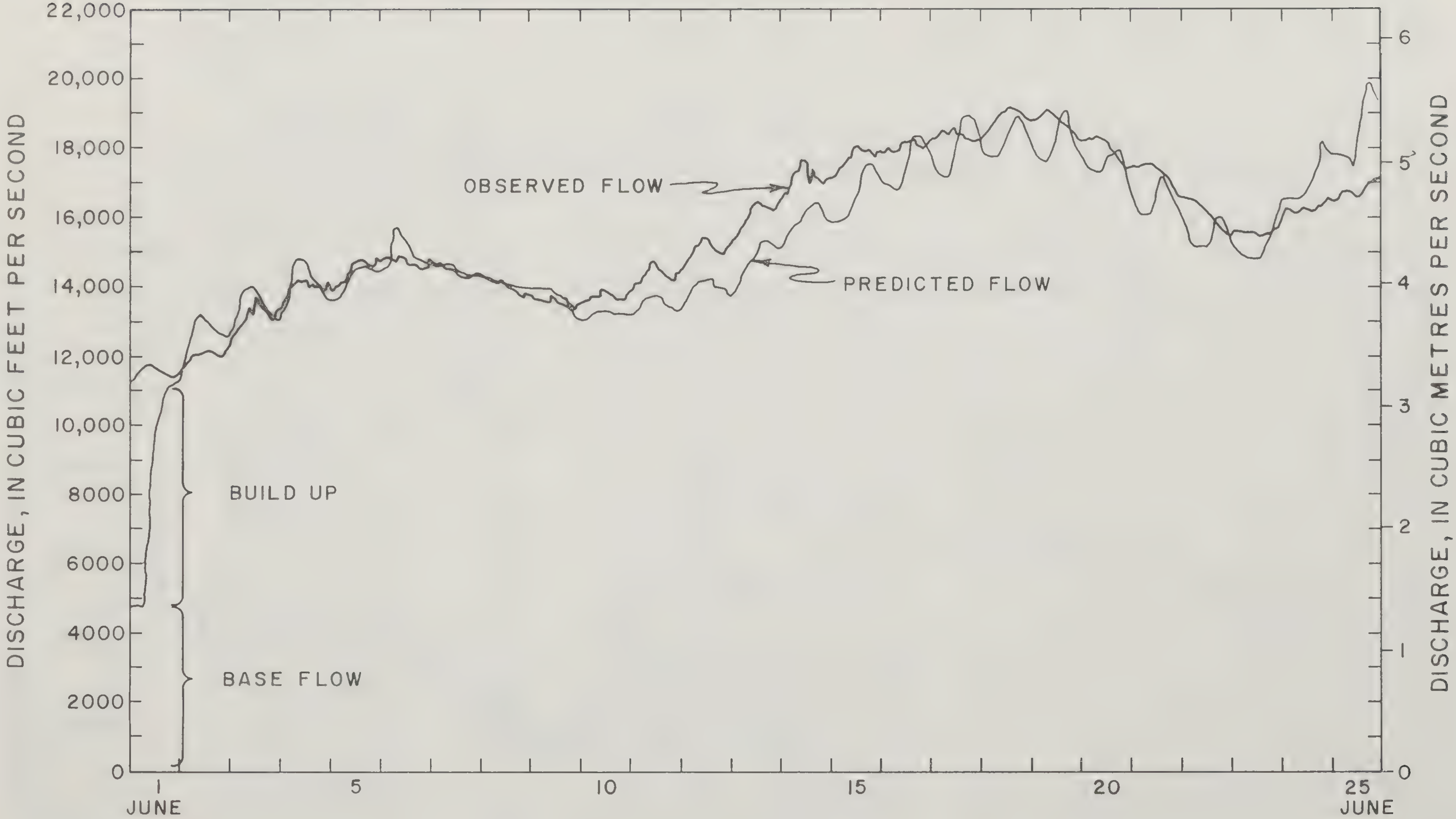


Figure 10.—Observed and modeled discharge hydrographs for the gaging station, Snake River at Wilson, Wyoming, June 1-25, 1974.

Extension of the Celerity-Discharge Relation

The calibration indicates the ability of the model to reproduce the normally experienced flow conditions, but this may be of little value when flows 80 times as large are found. A method was needed to determine how fast waves would travel at the large discharges produced by the dam break. This was done as follows:

First, cross sections thought to be representative of each routing reach were selected. Elevations for cross sections were determined from U.S. Geological Survey 1:24,000-scale topographic maps with 20-ft (6-m) contours. Rating curves of discharge versus stage were developed for these sections using the Manning equation

$$Q = \frac{1.486}{n} A R^{2/3} S^{1/2}$$

where Q is discharge, n is a roughness parameter, A is the cross sectional area, R is the hydraulic radius, and S is the slope of the energy gradient. In this use, S was assumed to be equal to the channel bed slope over the length of the reach in question. The roughness coefficient, n, was determined from field observations. The seven cross sections used in this report and the corresponding discharge rating curves are shown in figures 11-17. From these rating curves, the celerity-versus-discharge relation was determined as

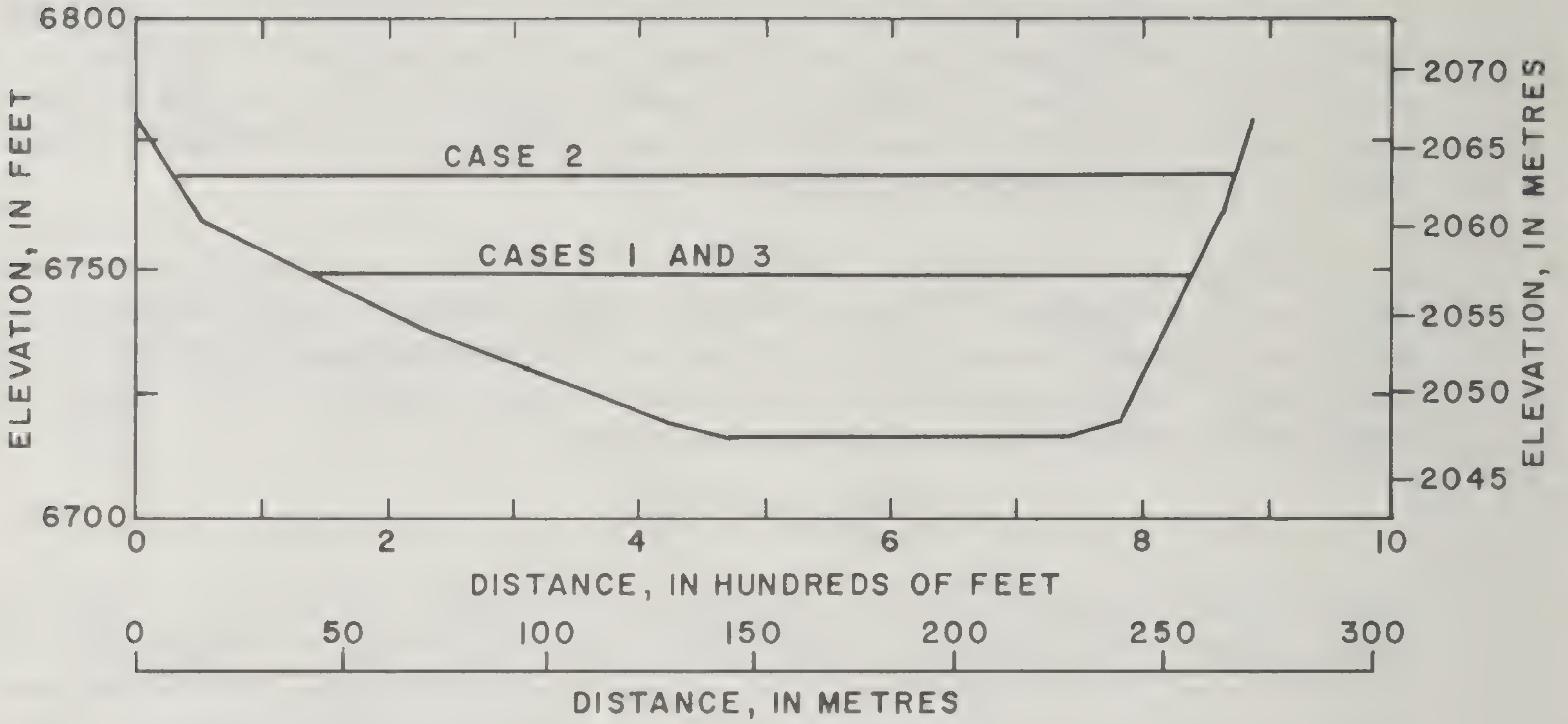
$$C = \frac{1}{W} \frac{dQ}{dY}$$

where C is celerity, W is the channel width, and Y is depth. This concept is described by Chow (1959) in Part V on unsteady flow.

RESULTS OF STREAMFLOW ROUTING

The model was used to determine the discharge at four of the cross sections (1, 3, 5, and 7) due to the rupture of the dam. These were located 3.8, 12.0, 21.12 (Moose), and 33.94 (Wilson) mi (6.1, 19.3, 34.0, and 54.6 km) below the dam. Intermediate cross sections (2, 4, and 6) were used to help define a flood inundation map (plate 1) and the flow characteristics. For each case, the model was run in four steps to determine these discharges as follows:

First, the dam-break wave was routed to cross section 1. To account for Pacific Creek, 2,500 ft^3/s (71 m^3/s) was added to the results. Second, an additional 5,000 ft^3/s (142 m^3/s) was added to the flow at cross section 1 to account for Buffalo Fork, and the results were routed to cross section 3. Third, the flow at cross section 3 was routed to cross section 5 (Moose). Fourth, the flows at cross section 5 were routed to cross section 7 (Wilson), and 4,000 ft^3/s (113 m^3/s) was added to account for inflow from the Gros Ventre River. The amounts added at the various stages were determined by inspection of historical records for Pacific Creek and Buffalo Fork and by estimation for the Gros Ventre River.



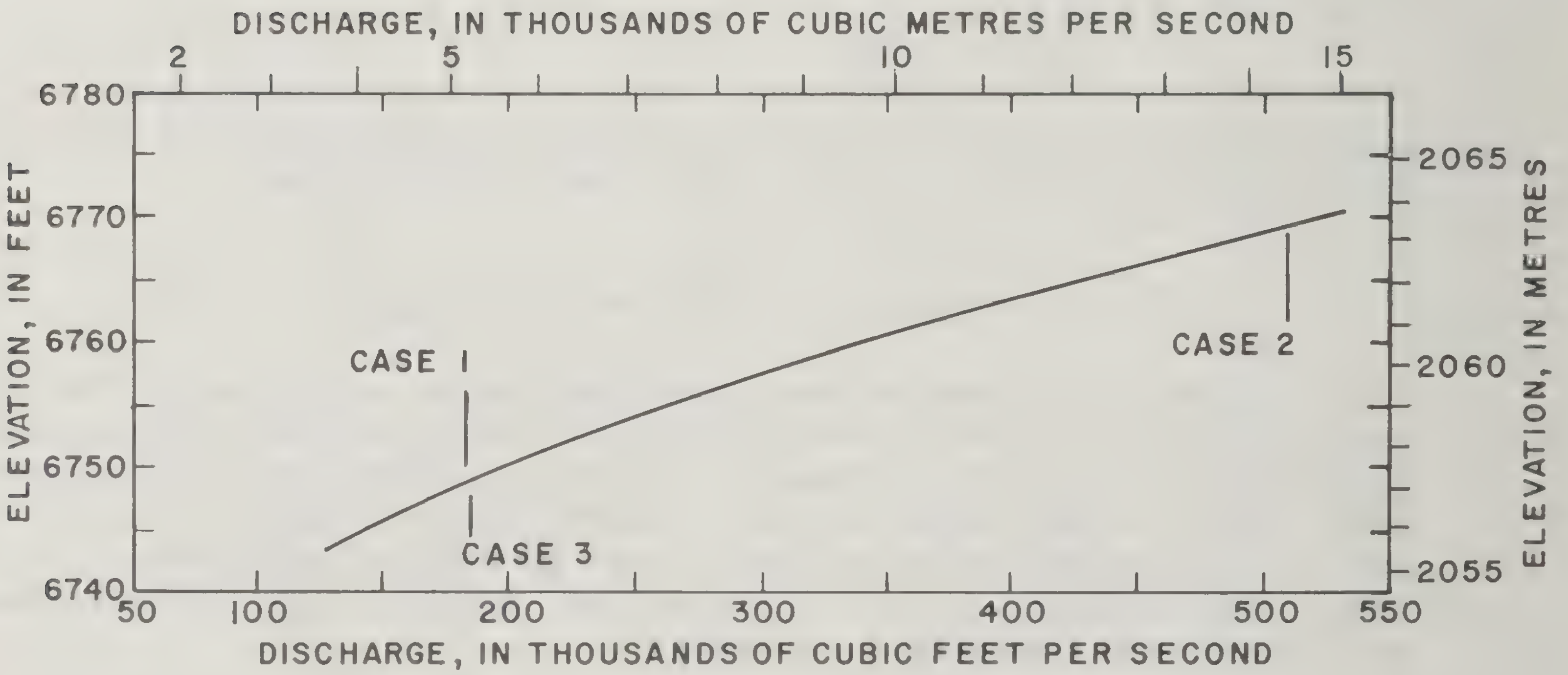


Figure 11.—Channel cross section 1, showing water-surface elevations for each of the three floods and corresponding discharge-elevation curves. Datum is mean sea level.

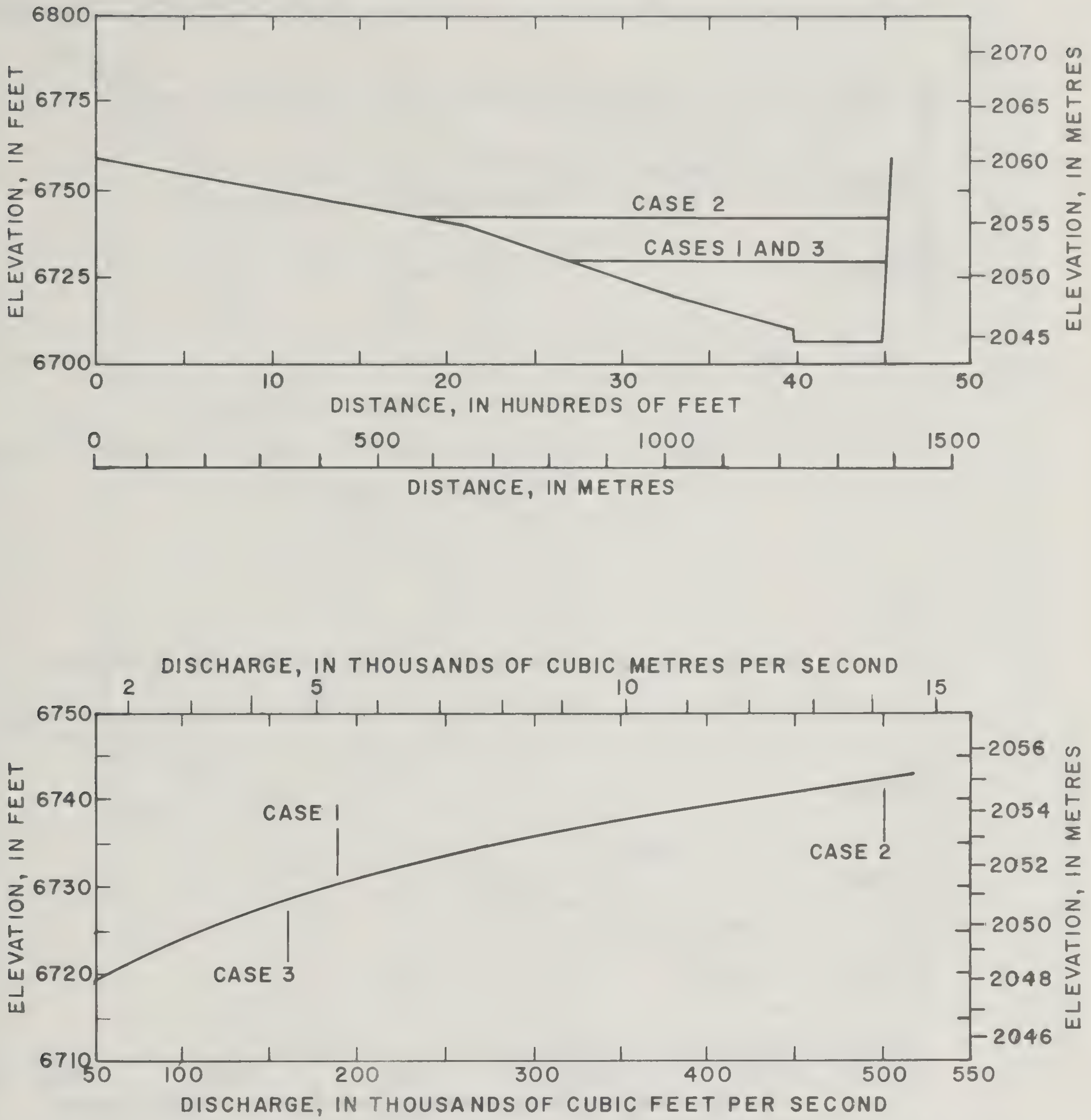


Figure 12.—Channel cross section 2, showing water-surface elevations for each of the three floods and corresponding discharge-elevation curves. Datum is mean sea level.

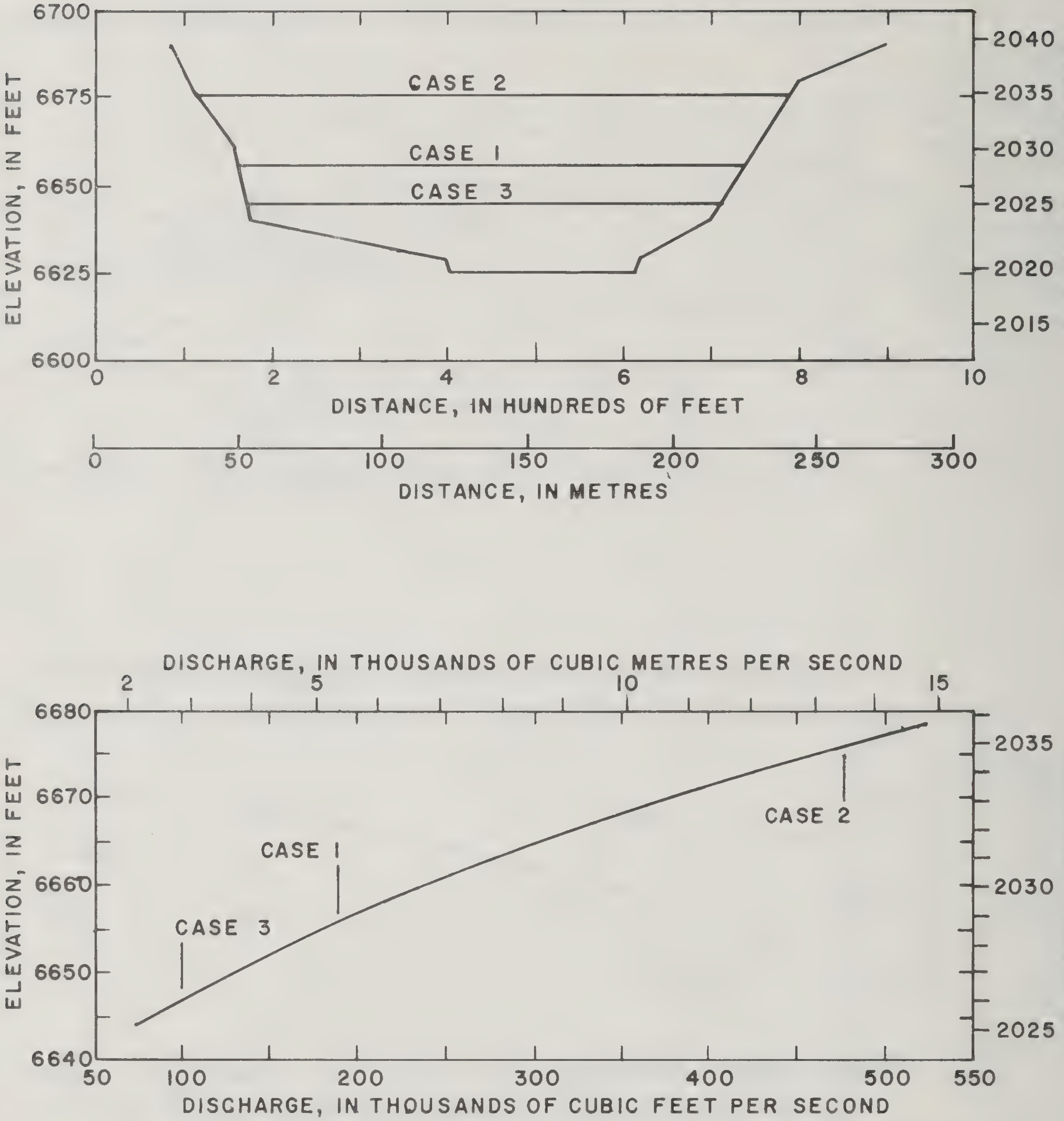


Figure 13.—Channel cross section 3, showing water-surface elevations for each of the three floods and corresponding discharge-elevation curves. Datum is mean sea level.

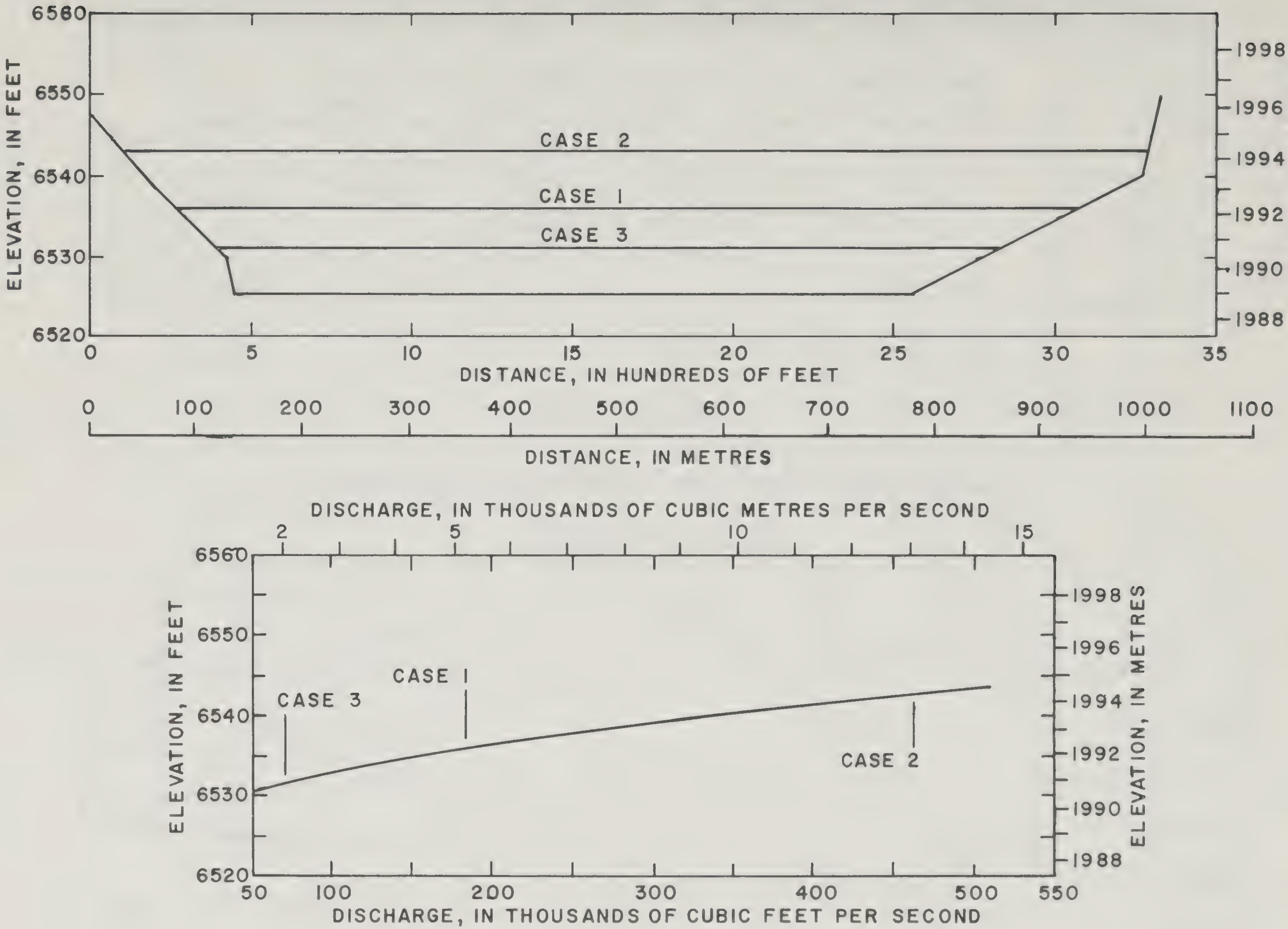


Figure 14.—Channel cross section 4, showing water-surface elevations for each of the three floods and corresponding discharge-elevation curves. Datum is mean sea level.

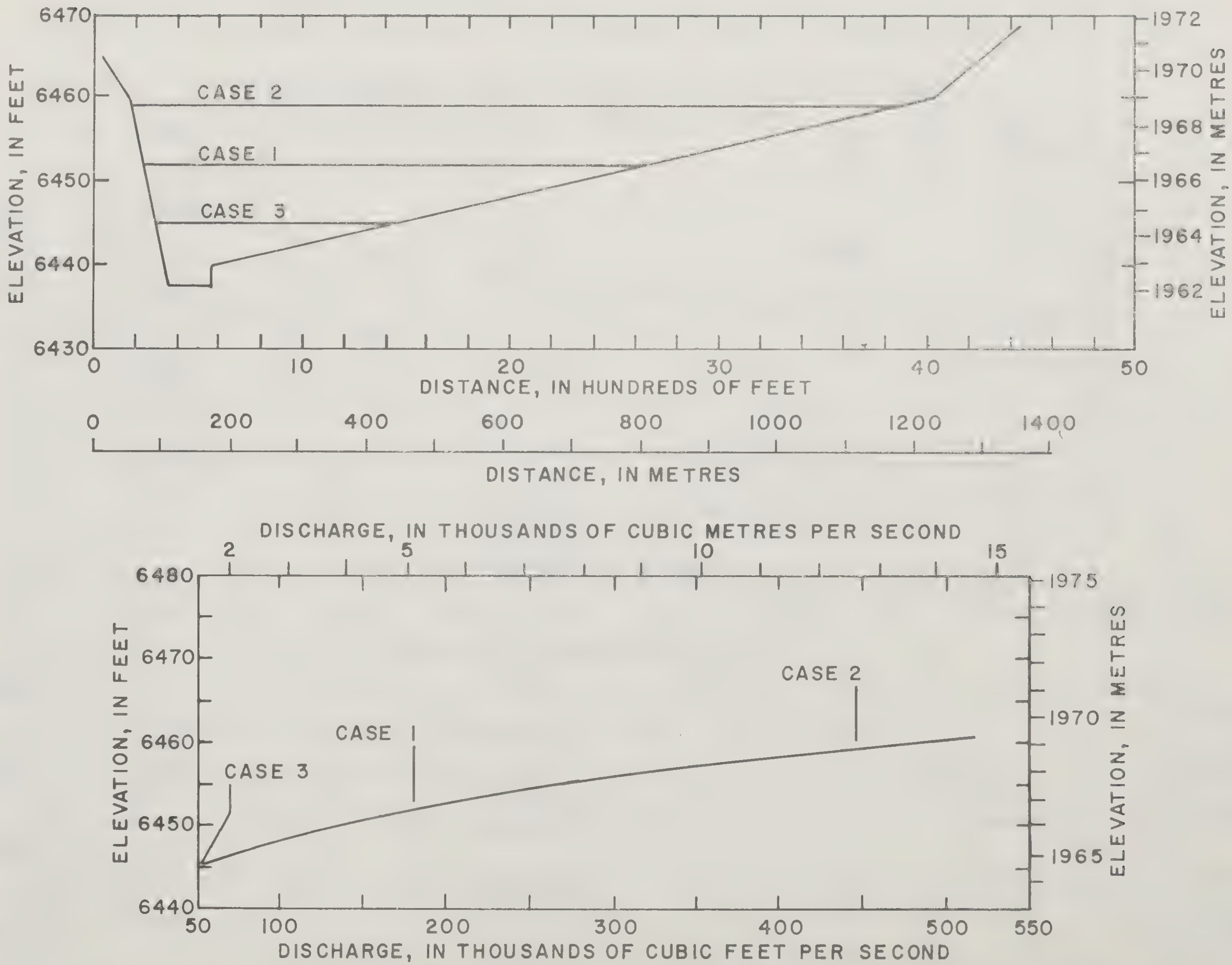


Figure 15.—Channel cross section 5, showing water-surface elevations for each of the three floods and corresponding discharge-elevation curves. Datum is mean sea level.

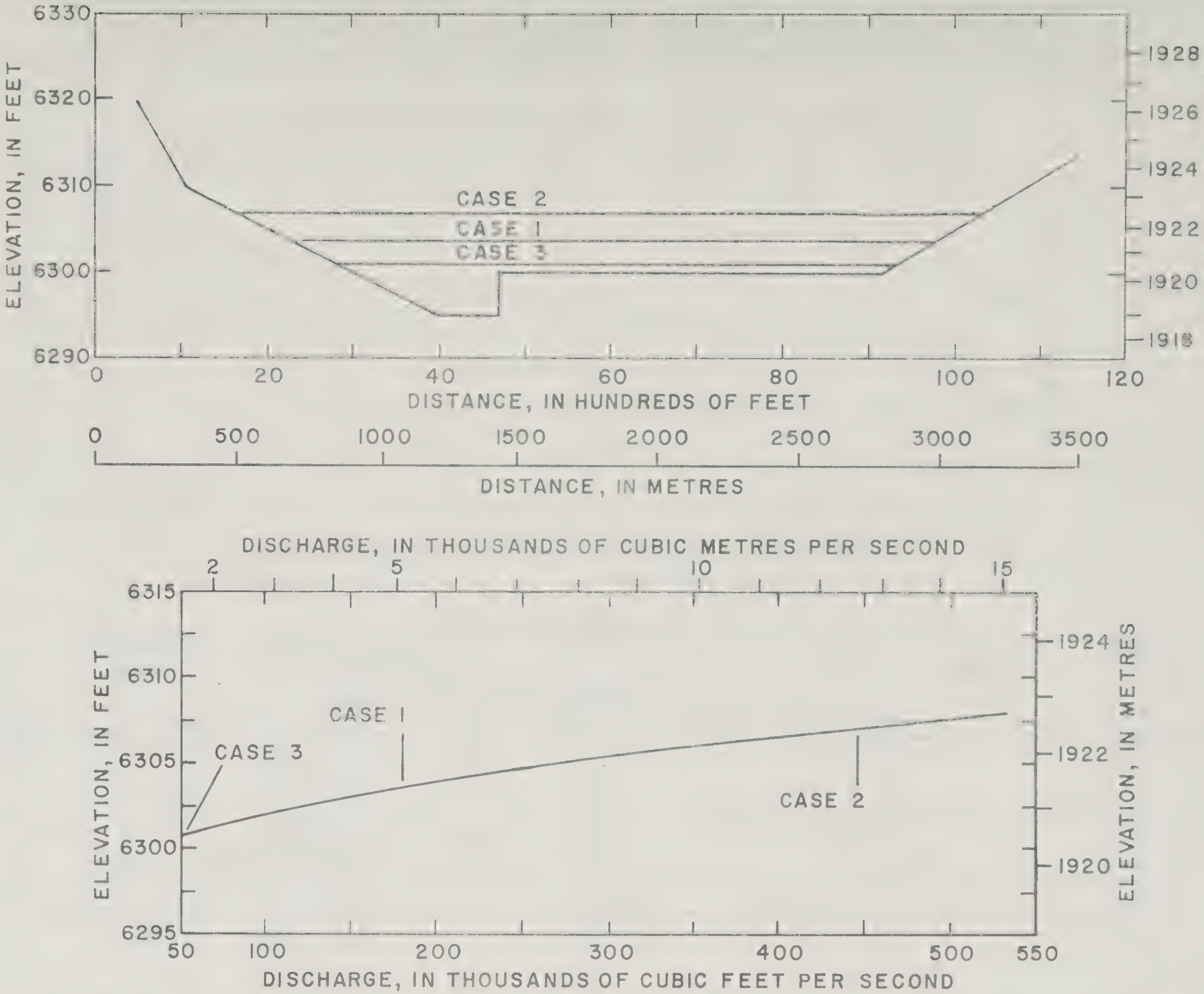


Figure 16.—Channel cross section 6, showing water-surface elevations for each of the three floods and corresponding discharge-elevation curves. Datum is mean sea level.

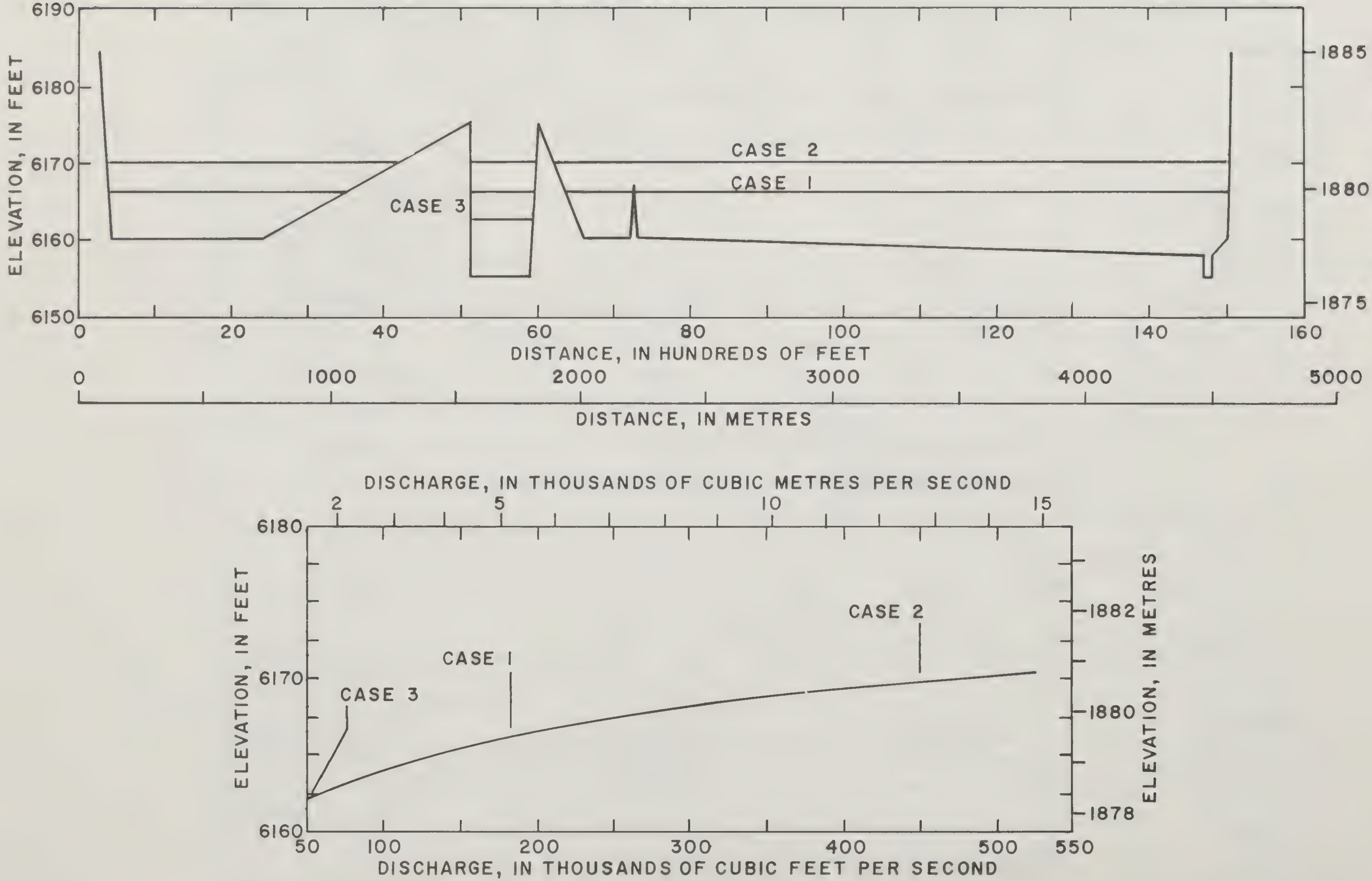


Figure 17.—Channel cross section 7, showing water-surface elevations for each of the three floods and corresponding discharge-elevation curves. Datum is mean sea level.

The results of the routing for the three cases considered are shown in figures 2, 3, and 4. With the exception of case 3, the movement of the waves is primarily translational with little reduction in discharge (10-15 percent between the dam and Wilson). In case 3, the wave overtopping the dam, the attenuation is more significant (more than a 5-fold decrease) due to the short duration of the wave and the smaller quantity of water involved.

FLOOD-INUNDATION MAPS

Water-surface elevations at the cross sections were determined from the rating curves for the discharges previously obtained from the streamflow-routing model (figs. 3, 4, and 6). Discharges for cross sections 2, 4, and 6 were interpolated between routed discharges at cross sections 1, 3, 5, and 7. Water-surface elevations at the seven cross sections were used to plot longitudinal profiles (fig. 18). As can be seen from figure 18, the slope of the water surface increases downstream from the two constrictions at section 1, 3.8 mi (6.1 km) below the dam, and at section 3, 12.0 mi (19.3 km) below the dam, and is nearly uniform for the rest of the routed distance.

Inundation boundaries, as determined from the cross sections and the water-surface profiles, were drawn on the 1:24,000-scale topographic maps for the three cases and then transferred to the final map with a scale of 1:62,500 and 80-ft (24-m) contours (plate 1).

The western boundary delineated on the inundation map between cross sections 6 and 7 may be somewhat inaccurate due to the flatness of the valley between the Snake River and Fish Creek.

The case 3 flow was shown as confined between the levees from about 24.5 mi (39.4 km) below the dam to cross section 7 at 33.9 mi (54.5 km) below the dam. If the levees are not as well constructed upstream as they are at cross section 7 (which has sufficient capacity to handle the case 3 flow), the flow may spread across the valley beyond the levees.

Graphs of mean velocity versus water elevation were plotted, based on discharge-elevation and area-elevation relations, for each cross section. Mean velocities for maximum discharge for each of the three cases of floodflow were obtained from the graphs and are listed in table 1, in both ft/s and m/s.

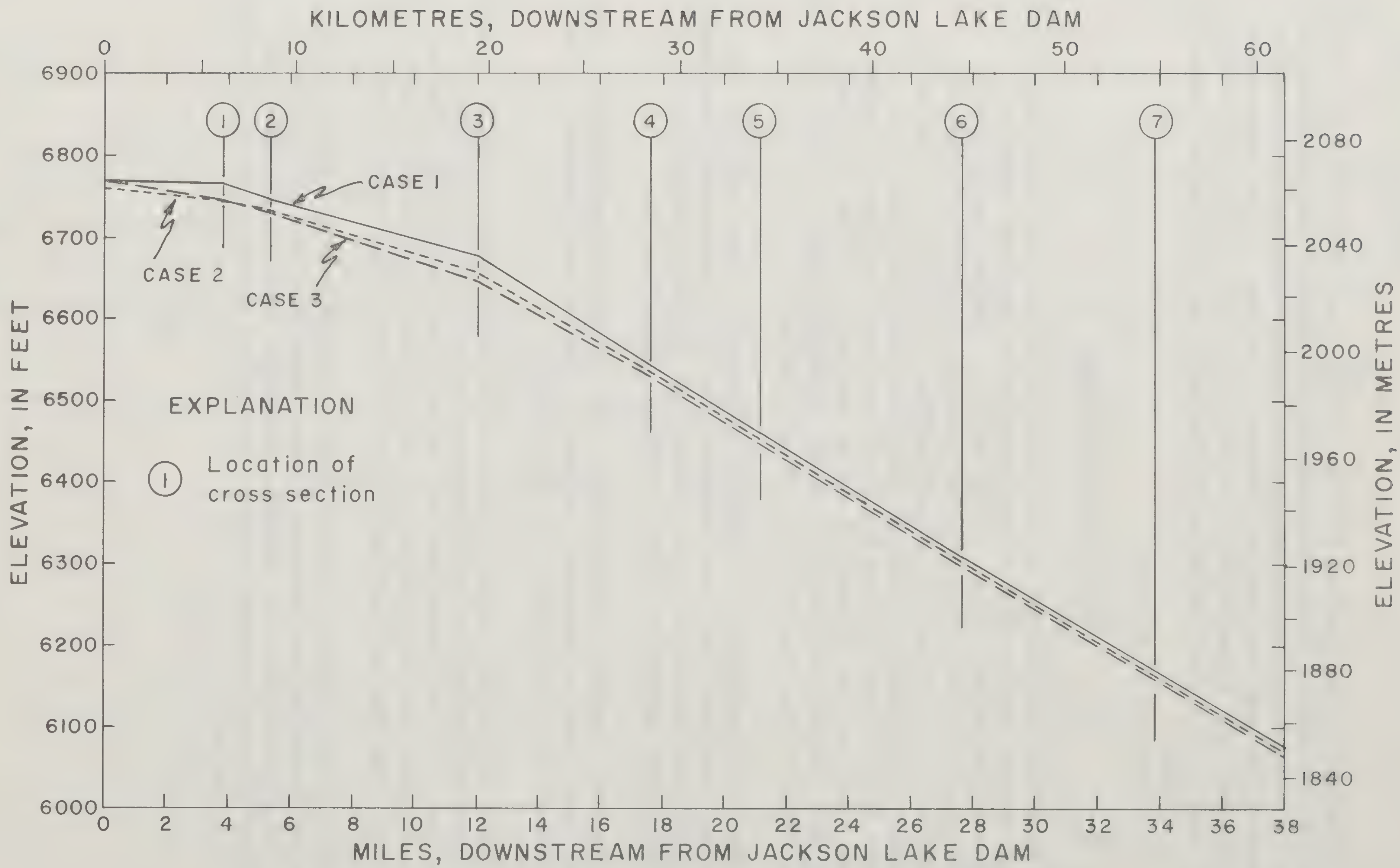


Figure 18.—Longitudinal profiles of the maximum water-surface elevations for the three floods. Datum is mean sea level.

Table 1.--Mean velocities at cross sections.

Cross section	Case 1		Case 2		Case 3	
	ft/s	m/s	ft/s	m/s	ft/s	m/s
1	11.3	3.4	15.3	4.7	11.3	3.4
2	7.2	2.2	9.3	2.8	7.0	2.1
3	14.2	4.3	18.3	5.6	11.3	3.4
4	7.5	2.3	10.3	3.1	5.7	1.7
5	10.0	3.0	11.1	3.4	9.3	2.8
6	7.4	2.3	8.7	2.7	5.7	1.7
7	8.1	2.5	8.8	2.7	7.3	2.2

OTHER POSSIBLE EFFECTS OF FLOODS

Channel Scour and Fill

Significant scour and fill would occur as a result of flows of the magnitude described in this report. The unconsolidated alluvium, deposited in the river valley by glacial-fed streams, is readily susceptible to movement. Typical of streams formed in glacial-outwash areas, the present channel is braided and unstable, subject to sloughing of the banks and bed movement. The abrasive effect of the flow would be amplified by the movement of debris. Sand and gravel would be scoured from constricted sections and deposited in areas where flow velocity had decreased enough so that it would no longer support the suspended sediment or cause bed movement. Leopold and others (1964) discussed several examples of river-bed scour during floods, including the variations of scour and fill on the rising and falling limbs of the hydrograph and variations over long reaches of the river.

With respect to channel changes, Quick (1974) describes floods as short-duration, high-energy events that try to superimpose a new channel pattern over the meander patterns that were formed by previous long-duration events. The peak flows would tend to cut across the meanders and temporarily straighten the channel. During the long recession of the flow as the reservoir drained, deposition of sediment and a re-emergence of channels would take place, with the possibility that the channel could change location throughout the period. E. V. Richardson and D. B. Simons, of Colorado State University, and C. F. Nordin, of the U.S. Geological Survey, presently (1974) are studying channel changes caused by major floods and by regulation of flow in both the Snake and the Gros Ventre River basins. Their studies indicate that the Snake River has shifted its channel in the past and that floods of the magnitudes indicated in this report would significantly change the present channel (E. V. Richardson, written commun., 1975).

Predictions as to the exact location of the main streambed after passage of the hypothetical flows caused by the dam failure are somewhat uncertain. Love and Reed (1968) suggest that the Snake River, if it were not confined by the levees, would move to the west toward the Teton Fault zone and the lower elevation of Fish Creek.

Due to the flatness of the valley, the area downstream from cross section 6 is especially susceptible to channel changes. Present aggradation in some sections of the confined channel also increases the possibility of channel changes in this area. At a point about 0.7 mi (1.1 km) downstream from cross section 6, the elevation of Lake Creek is lower than that of the Snake River. The relative elevations of the streams at this point indicate this area to be one of probable river change, possibly approximating the course of Lake Creek if there is overflow of the main channel levees. Lake Creek and intermittent streams in this area tend to drain toward Fish Creek, which might indicate a future route of the Snake River. Further tilting of the valley could also tend to move the flow toward Fish Creek.

Contamination of Wells

Wells in the inundated areas would become contaminated with suspended sediment and bacteria if flood water is able to get into the wells. Although wells in the inundated areas range up to a few hundred feet in depth, most are generally about 50 ft (15 m) deep. The wells tap alluvium. Water in Jackson Lake has low dissolved solids concentration. Analyses of water samples collected below Jackson Lake Dam June 4, 1971 and October 17, 1972 indicated 92 and 91 milligrams per litre of dissolved solids , respectively, (U.S. Geological Survey, 1973). The contamination of the wells would come from the bacteria and the sediment that the floodflow would pick up in its course downstream from Jackson Lake. The locations of wells in the study area, together with water-level and water-quality data, are included in a report by Cox (1974).

CONCLUSIONS

The flood-inundation map (plate 1) is the principal result of the study. The area between the inundation boundaries, as drawn on the map, would be flooded by the hypothetical flows selected for the study, according to the assumptions and analytical approach used. It is emphasized that these are hypothetical floods. While the probability of such floods occurring in the manner described is unknown, it is undoubtedly very small. If a large earthquake were to occur in the area, a resulting flood, if any, would not necessarily be the same magnitude as the hypothetical floods described nor would its behavior be identical.

Inundation as caused by the three cases of floodflow from Jackson Lake described in this report would be fairly well restricted to the present Snake River channel and flood plain in the upper reaches but could extend as far west as Fish Creek and as far east as Spring Creek in the lower reaches of the report area near Jackson.

Destruction of buildings and flood-plain vegetation would be expected in areas where velocity, depth of flow, and debris movement are significant.

Channel changes are inevitable due to the consistency of the streambed and the magnitude of the floods. Areas of probable channel changes could be denoted from topographic features, but prediction as to the exact relocation of the Snake River channel cannot be done with certainty.

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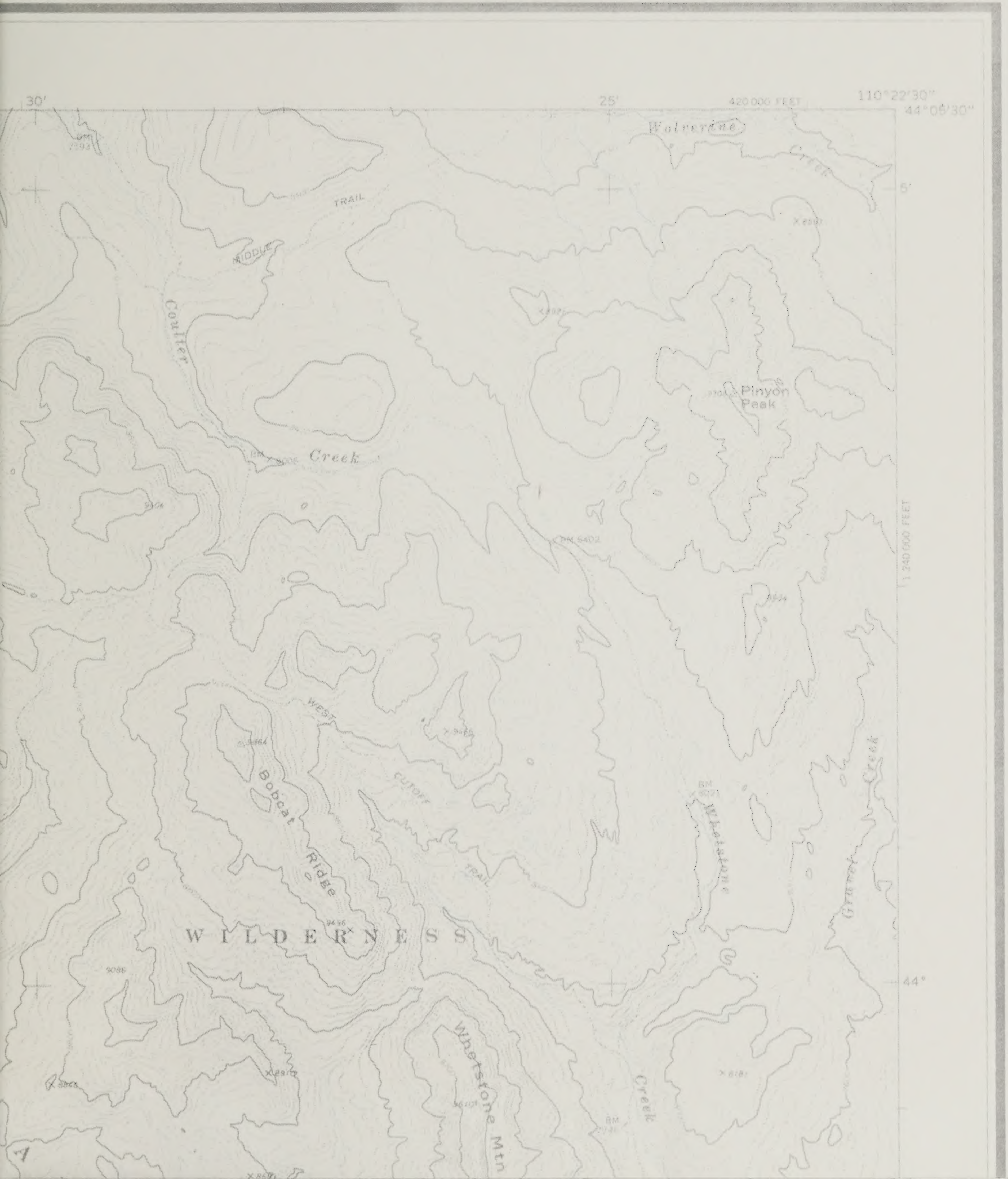
WATER-RESOURCES INVESTIGATIONS 76-77
OPEN-FILE REPORT
PLATE I



DEPARTMENT OF THE INTERIOR
UNITED STATES GEOLOGICAL SURVEY

PREPARED IN COOPERATION WITH THE
NATIONAL PARK SERVICE

WATER-RESOURCES INVESTIGATIONS 76-77
OPEN-FILE REPORT
PLATE I

UNITED STATES
DEPARTMENT OF THE INTERIOR
GEOLOGICAL SURVEY

FLOOD-INUNDATION MAP

WYOMING-TETON COUNTY

1968

N4326.5-W11022.5/39X35

SCALE 1:62500

CONTOUR INTERVAL 80 FEET
DOTTED LINES REPRESENT 40 FOOT CONTOURS
DATUM IS MEAN SEA LEVEL
JACKSON LAKE DEPTH CURVE INTERVAL IS 25 FEET-DATUM IS LAKE LEVEL 6772

Mapped, edited, and published by the Geological Survey

Control by USGS and USC&GS

Compiled from 1:24,000-scale, 7.5 minute maps, and 1:62,500-scale, 15 minute maps, dated 1956, 1959, 1963, 1965, and 1968. Partial cultural revision 1970. Supersedes map dated 1948.

Polyconic projection. 1927 North American datum
10,000-foot grid based on Wyoming coordinate system, west zone

Where omitted, land lines have not been established

Base Grand Teton National Park, 1968.

EXPLANATION

▲ Gaging station

(6)——(6) Cross section location and number

FLOOD-INUNDATION BOUNDARIES

Case 1

Case 2

Case 3

Inundated area, cases 1, 2, and 3.